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CCXX.

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WIND PRESSURE UPON BRIDGES.

By C. SHALER SMITH, Member A. S. C. E.

READ DECEMBER 15TH, 1880.

WITH DISCUSSIONS

By CHARLES DOUGLAS FOX, ROBERT E. JOHNSTON, G. BOUSCAREN, WILLARD S. POPE, JOSEPH M. WILSON, A. GOTTLEIB, D. J. WHITTEMORE, A. S. C. WURTELE, CHARLES A. SMITH, ROBERT FLETCHER and O. CHANUTE.

It may be safely stated that the practice among American engineers is very nearly uniform in regard to wind strains. My own specifications, which I have used in constructing a number of exceptionally high bridges, some of them in especially exposed positions, are as follows:

[NOTE.—The substance of this paper was embodied in a letter written in response to a request from the Secretary of the American Society of Civil Engineers. The occasion of this request was to obtain a statement of the practice of American engineers in regard to wind strains, and to secure this the Secretary corresponded with a number of the members of the Society who had large experience in the construction of bridges. Extracts from the letters of these members were transmitted to Sir John Hawkshaw, Hon. Member A. S. C. E., who had asked the Secretary to obtain this information for the use of a Commission upon the subject in England. With the consent of Mr. C. Shaler Smith, the substance of his letter is issued as this paper.—EDITOR.]

"The portal, vertical and horizontal bracing shall be proportioned for "a wind pressure of 30 lbs. per square foot on the surface of a train "averaging 10 square feet per lineal foot, and on twice the vertical sur- "face of one truss. The 300 lbs. pressure per lineal foot due to the train "surface shall be treated as a moving load, and the pressure on the trusses "as a fixed load. Trusses of less than 200 feet span shall also be propor- "tioned for a pressure of 50 lbs. per square foot when unloaded, and the "greatest strain by either method of computation shall in each case be "used in determining the sectional areas of the bracing.

"Iron piers, and spans carried by them, shall be designed to resist "a wind force of 30 lbs. per square foot on train and structure, or 50 lbs. "per square foot on the structure alone. The compressive strains on the "leeward columns of the piers shall be computed with the assumption "that the maximum load is on the bridge, and to these shall be added "the compressive strains produced by the wind, and the columns shall "be proportioned to resist these combined strains with a factor of safety "of four. The minus strains on the windward columns shall be com- "puted with the lightest train on the bridge which will not be blown off "by a wind force of 30 lbs. per square foot, and such a width of base "shall be given to the pier that there shall be no tension in any of the "columns composing it.

"If the bridge is on a curve, the centrifugal force due to the max- "imum load moving at forty miles per hour shall be added to the strains "arising from wind pressure on span and pier. To resist the strains "computed as above specified, tensile bracing shall be proportioned at "15 000 lbs. per square inch; the shearing sections of pins, bolts and "rivets, at 10 000 lbs. per square inch; rivets in tension at 5 000 lbs. "per inch; bending stress of pins at 22,500 lbs. per square inch, "and struts in compression with a factor of safety of four. The end con- "nections of all wind bracing shall be stronger than the bracing itself. "Joints of pier columns shall be fully spliced, so that not less than one- "half of the sectional area of the column is available for a tensile strain. "In case the sectional area of a pier column proportioned for the load, "wind and centrifugal strains combined, with a factor of four, is less "than the section required for load and centrifugal strains only, at the "specified factor for load stress, the largest section shall be used in all "cases.

"To all strains computed as above for wind bracing there shall be

"added an initial stress of 10 000 lbs. in each member, in order to allow "for the strains produced in screwing up the rods during the adjustment "of the bracing." * * *

Two features in the foregoing require explanation :

First—Relating to the area of exposed surface, my experiments on the Rock Island pivot span in 1872 showed that the resistance to wind by the structure was that due to rather more than 1 $\frac{8}{10}$ times the exposed surface of one truss.

Last—The wind pressure on the train is treated as a rolling load because the action of lateral shear on wind bracing is precisely the same as that of vertical shear on the web members, *i. e.*, greatest with partial loads.

The following railways and bridge companies use practically the same specifications as the foregoing :

Chicago, Milwaukee & St. Paul Railway.
Cincinnati Southern Railway.
Keystone Bridge Company, of Pittsburgh.
St. Louis, Kansas City & Northern Railway.
Denver & Rio Grande Railway.
Atchison, Topeka & Santa Fé Railway.
St. Paul, Minneapolis & Manitoba Railway.
Gulf, Colorado & Santa Fé Railway.

Many engineers prefer to express wind force in pounds per lineal foot of bridge instead of per square foot of exposed surface. Using a 200 feet span as an example, the specifications in question can be condensed as follows :

Fixed load in plane of roadway, 210 lbs. per lineal foot.
" " in plane of other chord, 130 lbs. per lineal foot.
Moving load in plane of roadway, 300 lbs. per lineal foot.
Iron in tension, 15 000 lbs. per inch.
Iron in compression; factor, 4.

The Erie Railway specifications are as follows :

Fixed load, roadway chord, 150 lbs. per lineal foot.
" " other " 150 " " " "
Moving load, roadway " 300 " " " "
Iron in tension at 15 000 lbs.
" " compression, factor 4.

The Greenville & Columbia Railway specifications are the same as

the Erie, except that 200 lbs. per foot are used for the top chord of through bridges.

The Louisville & Nashville specifications are the same as Erie, but no moving load is used—simply 450 lbs. per foot on roadway bracing and 150 lbs. per foot on the other chord.

The Lake Shore road uses 300 lbs. per foot fixed load on roadway laterals and 150 lbs. on the other chord, but strains in tension are 10 000 lbs. per square inch only.

The Chicago, Rock Island & Pacific Railway prescribes 300 lbs. per foot, with iron at 10 000 lbs. per inch.

The Pittsburgh, Cincinnati & St. Louis Railway requires 300 lbs. per foot for the train and 30 lbs. per square foot on one truss only.

The Louisville Bridge Company uses the same specifications as the Louisville & Nashville Railway.

For the bridge over the Missouri, at Glasgow, 50 lbs. per square foot on one truss and 300 lbs. per foot for train were used.

For the Eads bridge, at St. Louis, 50 lbs. per square foot on the structure alone was the specified pressure.

For the Kentucky River Bridge the wind pressure was assumed at 31½ lbs. per square foot on spans, train and piers, and factor four was used in proportioning the bracing.

The Portage Bridge, in New York, was built to resist 30 lbs. per square foot on structure and train, and 50 lbs. per square foot on the structure alone.

Both of these last bridges are unusually lofty and in exposed positions, and both are on iron skeleton piers.

The 520 feet span over the Ohio, at Cincinnati, was designed to withstand 50 lbs. per square foot on structure alone, or 30 lbs. per foot on train and structure combined.

Thirty lbs. per square foot was recommended by the majority of the members of the Committee on the "Means of Averting Bridge Accidents," in their report to the American Society of Civil Engineers in March, 1875. One of that Committee, however—General Ellis—advised 40 lbs. per square foot, which specification is also used by Mr. Slataper, Chief Engineer of the Pennsylvania Railway Company.

Having given the foregoing examples as a fair average of the current practice of American engineers in regard to wind pressure, I will add the data on which I base the opinion that the specifications first recited

fulfil all the requirements of the case, even in what is known as the tornado belt of our Western States.

For a number of years past, whenever practicable, I have personally visited the tracks of destructive storms as soon as possible after their occurrence, for the purpose of determining the maximum force and width of the path in each case.

The most violent on my records are as follows :

First.—East St. Louis, 1871 : Locomotive overturned ; maximum force required, 93 lbs. per square foot.

Second.—St. Charles, 1877 : Jail destroyed ; force required, 84.3 lbs. per square foot.

Third.—Marshfield, Mo., 1880 : Brick mansion house levelled ; force required, 58 lbs. per square foot.

Fourth.—Havre de Grace, Md., 1866 : Ten spans wooden Howe truss bridge, 250 feet each, blown over ; force required, 27 lbs. per square foot.

Fifth.—Decatur, Alabama, 1870 : Two spans of Combination Triangular truss blown over ; force required, 26 lbs. per square foot.

Sixth.—Meredosia, Ill., 1880 : One span wooden Howe truss, 150 feet long, overturned ; force, 24 lbs. per foot.

Seventh.—Omaha, Nebraska, 1877 : Two spans iron Post truss, 250 feet each, blown down ; force required, $18\frac{7}{10}$ lbs per square foot.

Also, sundry cases of train derailment caused by wind, the maximum force required being $30\frac{1}{2}$ lbs. per square foot.

In each of the foregoing cases I have given what appeared to be the maximum effort of the wind and the lowest pressure required to produce the observed result. It is therefore not unlikely that the real force of the wind in each example was greater than I have stated it. Some of the tornadoes were very destructive—the Marshfield one, for instance, having cut a swath 46 miles long and 1800 feet wide, and killed and wounded over 250 people. To the above cited instances may be added the Tay Bridge disaster, in which case $20\frac{4}{10}$ lbs. per square foot on train and bridge were required to destroy the piers, through the rupture of the vertical bracing in the four bottom tiers of the pier over which the train was passing when failure began. My reasons for considering 30 lbs. per square foot sufficient for a working specification, when the above record shows much higher pressures, are these—

First.—I very much doubt if a *direct* wind or gale ever exceeds 30 lbs.

per foot; whirlwinds do exceed it, but the width of the pathway of maximum effort in these is usually very narrow, although the general direction is so erratic that the appearance of the *debris* is generally such as to produce the impression that the vortex was much larger than was really the case. With the exception of the Marshfield tornado, I have yet to find a storm swath where the width of pathway wherein the force exceeded 30 lbs. per square foot, was more than 60 feet wide.

The St. Charles tornado is a case in point. This whirlwind cut a swath about 1000 feet wide for 14 miles, and destroyed over 300 houses, exerting a force of over 84 lbs. per square foot at its point of maximum effort. It crossed the middle span of the St. Charles bridge nearly at right angles, and developed a pressure of $52\frac{4}{10}$ lbs. per square foot in picking up and crushing a barrel of tar, which stood on the bridge in the path of the vortex. The width of the vortex was distinctly marked on the span by the circle in which the tar was spun around, the wreckage left upon it, and the points at which it ceased to destroy the flooring. This width was thus shown to be slightly over 60 feet, and, guided by this, I was subsequently enabled to locate the path traveled by the central vortex throughout the entire length of the storm swath. The bridge itself was uninjured, although it was only proportioned to withstand 30 lbs. per square foot, with a strain of 20,000 lbs. per square inch on the braces. This span was 320 feet long, 30 feet in depth, and the top chord was 120 feet above the water. I consider it very unlikely that a bridge of over 200 feet span will ever be exposed to a wind force of more than 30 lbs. per square foot, acting *in the same direction* over its entire length.

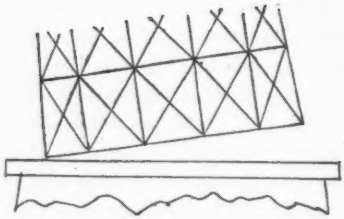
Next.—A fully loaded passenger train, and the heaviest possible freight train will leave the track at the respective pressures of $31\frac{1}{4}$ and $56\frac{1}{2}$ lbs. per square foot. If the braces are proportioned at 15,000 lbs. per square inch, with a wind pressure of 30 lbs. per square foot, they will still be within their limit of elasticity at the moment when the train is blown from the track in either case. Destruction of the span will then take place, if at all, from the effects of derailment; to resist which greater strength in the wind bracing will be of no value.

Next.—If there is no tension in the pier columns until 30 lbs. wind pressure is reached, and these columns are properly spliced and anchored, as per specifications, there will be an ample margin of tensile strength in any case where this pressure may be exceeded.

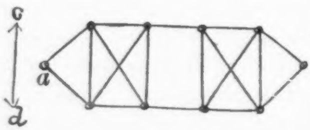
Last.—In view of the comparative rarity of these extreme strains and the consequent slight fatigue to which the iron is exposed, the high stresses imposed on the wind bracing are perfectly legitimate.

It will be noted that the specification as to width of base being such as to eliminate all tensile strains in the pier columns at the specified wind pressure, marks a characteristic difference between European and American practice. The French, Swiss and Austrian viaducts are all exposed to tension in the piers at comparatively low wind pressures, and, although these strains are provided for in the design of each structure, this provision includes subjecting cast-iron to tension, something not permissible in American practice, although our best cast-iron is much superior to that of Europe. Indeed, I am not aware of any skeleton bridge piers in this country which have been built within the last ten years having cast-iron pier columns. Another radical difference is to be found in the structural design of viaduct piers, which in Europe are generally composed of from six to fourteen columns, including two columns as wind braces, one at each end of the pier, while in America the material is concentrated in four or eight columns; the wind braces and load carrying columns being the same. As the internal bracing is or should be strong enough to enable the pier to turn over without collapsing, the great weakness of any pier, which must rotate on a single point, is readily seen from the following sketches:

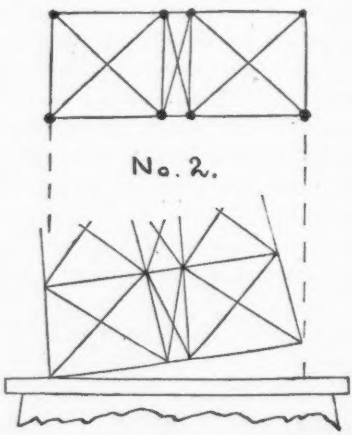
Sketch No. 1 shows the ordinary European ground plan of a skeleton pier, and the position which the pier must assume, if the internal bracing is strong enough to hold the pier together until the overturning pressure is reached. It will be seen at once that collapsing must result from the impossibility of maintaining column "*a*" in the direction "*c d.*"



No. 1.



Sketch No. 2 shows an American pier of the usual type, and the fact that collapsing can be easily guarded against becomes apparent on inspection. It is also evident that unless collapsing is prevented, the full value of the anchorage bolts cannot be obtained. In my enumeration of various railway specifications I have given one from each representative district of the country, excepting New England, and the structures mentioned are, with one exception, located in what is called the "tornado area" of the Western States.



No. 2.

The following account of the St. Charles tornado was written for the *Cosmos*, a local journal of date of March 1st, 1876, by a personal observer, Mr. C. C. Davis, and is published as giving, in the opinion of Mr. C. Shaler Smith, an accurate description of the commencement of the whirl:

At half past one o'clock, on last Sunday afternoon, a heavy storm cloud was observed directly south of the city, moving slowly due north. A few minutes later a similar cloud, but at a greater elevation, and with a much more rapid motion, appeared on the horizon, a few degrees south of west, its course being nearly east. The surface wind at first was light and from the southeast. The clouds soon lapped. Discharges of electricity from the lower to the upper cloud became frequent, but not heavy, and with their junction the entire mass rapidly settled towards the earth. Patches of flying scud appeared moving in every direction, and small showers of rain or hail could be seen falling at various points. The storm nimbus, by this time, had covered all the upper part of the city, its normal cloud tint being here and there flecked with dissolving spots of an unearthly green. Soon came signs of fury which was to follow. The wind came in light fitful gusts from every direction, succeeded by intervals of dead calm, the ominous stillness of the latter being especially marked. After a few short, sharp dashes of hail, interspersed with rain. a most singular phenomenon occurred : a genuine shower of snow balls, some of which, as they lay on the ground, could hardly be covered by a teacup. These were immense hail stones, generated in the cold upper cloud, and softened in passing through the warm and water-laden cloud from the south. So large were they that thei. white shapes were seen several hundred feet in the air—their color placing them in bold relief againt the murky sullen blackness of their parent cloud. A few moments of oppressive silence followed, when suddenly the rival storm-fiends leaped into infernal life, the ragged wind-scud flew in all directions, and almost instantly the lurid brown of a cyclone column appeared in the southwest, about three miles from the upper end of the city. Moving eastward, at first, until the river slopes had been reached, the terrible cloud column followed them to the city, giving dreadful presage of its coming in a deep-toned humming roar, very similar in sound and disagreeable effect to the noise and earth-shaking properties of the steam-blower of a locomotive engine, but magnified a hundred-fold. In appearance it was the true cyclone cloud—a brown, murky, vertical,

column, largest at its junction with the parent clouds—never more than 400 feet in diameter—reaching sometimes to the earth, where it destroyed all that it touched; and occasionally receding upwards, and passing harmlessly over those intervening spaces, where its powers were expended in the air above the buildings. Deliberately, but erratically, it moved through our devoted city—death in its roar, and destruction in its touch. The central vortex reached the spire of the German Methodist Church—a twist, a crash, and the spire went spinning upwards, but preserving its vertical position until near the top of its flight, when it was suddenly reversed, and came down point foremost. This vortex also touched the jail, the roof of which, together with the whole upper story, was twisted off—and to this hour the roof has not been found.

From this point the scene beggars all description. The total destruction of everything grasped by the tornado can be compared only to the wide-spread ruin produced by the explosion of a powder magazine. The streets were filled with heavy timbers reduced to the merest splinters. Walls, where not laid flat, were in many cases punctured with holes by flying joists and bricks, projected with a force not much less than if shot from a cannon. Near the vortex houses were destroyed by twisting and bursting; persons inside found it impossible to open the doors to get out, owing to the external vacuum and the pressure from within. On the circumference of the cyclone houses were crushed and overturned by the external force, and those living in this part of its track had doors and windows blown in from without, as a preliminary to the destruction of the building.

After crossing the railroad track, the column became less in diameter, followed a very devious route, and moved so slowly that one observer, who had seen it pass when he was near Morgan street, succeeded in repassing it while it was destroying the houses between Second street and the river in Frenchtown, and reached the bridge sometime before the tornado had finished its work of destruction on the west side of Second street.

After having followed the two main streets of the city to within three hundred feet of the bridge, the cyclone turned suddenly eastward, moved parallel to the bridge, and then crossed it at the middle of span four. It was an anxious moment to the three bridge men who were watching it. The whirlwind had destroyed everything it had touched

thus far, and the deadly column was black with timber and other *debris* of the houses it had wrecked—while high above the rest sailed a large roof, moving with terrific speed towards the bridge, which it was now clear would be crossed by the very vortex of the storm. For a second the cloud obscured the span, the flying roof soaring fifty feet above it, but in another instant the grand old bridge loomed out clear and sharp—its outlines all unbroken against the black background of the defeated Storm-King who swept muttering away.

From the bridge the column crossed to the St. Louis shore of the river, destroyed a number of large trees on that side and then turned and crossed to the west side again, raising the water nearly 200 feet high in its path, and reaching the St. Charles shore some two miles below the bridge. From this point it ceased to be a St. Charles institution and we must rely on our country correspondents to tell us of its doings beyond our immediate ken.

At 15 minutes past two the storm was over, and half an hour later the sun was shining, and the uncomfortable warmth of the morning had given place to a cold and chill atmosphere and western wind, which followed in the track of the higher and colder cloud, and which, in this battle of the elements, has proved victorious.

From the circular motion of the wind, it is difficult to measure its exact force, although there are several definite points where we are enabled to fix positively that the pressure exceeded a certain limit. One of these is the water tank at the jail. This stood fairly where the wind could act both on the sides and bottom. With the water in it at the time, we are told that $69\frac{9}{10}$ pounds per square foot upward and lateral force was required to over turn it, and it *was* overturned.

Another point fixed is the overturning force required to carry off a barrel half full of coal tar which stood on the bridge, and which was blown over and away. This needed $71\frac{5}{10}$ pounds pressure to overturn it. The significance of these figures will be understood when we state that a wind, with a speed of 80 miles per hour—giving a pressure of $31\frac{1}{2}$ pounds per square foot, is defined by the authorities as a "violent tornado." We are perfectly satisfied with our experience in cyclones and want no more of them.

DISCUSSIONS ON THE PAPER UPON WIND PRESSURE UPON BRIDGES.

CHARLES DOUGLAS FOX, of London, Corresponding Member A. S. C. E.—Mr. C. Shaler Smith's paper is to me very interesting as being the first instance I have seen of an attempt to formulate distinct rules upon this subject. It is understood that the French engineers adopt a maximum wind pressure of 55 pounds, but in Great Britain the matter has been hitherto left to the discretion of individual engineers, the Board of Trade requirements for railways making no mention of the point.

It is but comparatively rarely in English practice that bridges occur of such span and height as to render necessary any separate consideration of the wind pressure upon either girders or piers, the strength and stability necessary to resist the effects of trains running at high speed, with a factor of safety of 4, being, in ordinary cases, such as to provide a large margin for safety against the effects of wind. A perusal of the evidence given before the commissioners appointed to report upon the accident to the Tay Bridge will show that engineers and scientists also widely differ as to the pressure to be expected, both as to their force and their extent, ranging upon the structure in question, having spans of 245 feet, and a height of 115 feet, from 15 pounds up to 50 pounds per foot. The Astronomer Royal of England stated, in his evidence, his opinion, that, whilst a force of 40 or 50 pounds per superficial foot might be experienced upon limited areas, say up to 250 feet in width, the maximum average force upon a larger area of say 1 600 feet in width would, probably, not exceed 10 pounds per superficial foot, but this would, from Mr. Smith's paper, appear to be contrary to experience in the United States at least. It was further stated by engineering witnesses, that, leaving out of consideration the strength of the bolts which secured the cast-iron columns to the piers, it would have required a force of from 33 to 34½ pounds per foot, or, taking the bolts into account, then from 60 to 70 pounds per foot upon the piers, girders, and a train of carriages to overturn the bridge, if no failure took place in the details of construction. At the time of this inquiry, in the spring of 1880, no reliable measurements of pressure up to 50 pounds per foot had been made at the Royal Observatory, but I understand that, in the terrific easterly gale of January 18th last, the anemometer, on three occasions, registered

this amount of pressure. As, however, the instrument is placed on the Observatory Building, it is possible that it may be subjected to eddies, which would cause it to register more than the normal pressure.

My practice, which I believe, accords with that of many engineers in this country, has been, in dealing with roofs, to assume a maximum pressure per superficial foot of 40 pounds, of which 30 pounds are taken for the resultant pressure of the wind, at right angles to the roof, and 10 pounds for snow, it being very rare for much snow to lie upon such structures, when there is a heavy wind. For India, where our structures are exposed to severe cyclones, the same maximum pressure of 40 pounds is taken, in this case wholly for the wind, snow being unknown. The maximum strain upon the wrought iron of the roofs under such pressure would be about 16 000 pounds per sectional inch, giving a factor of safety of at least 3. Roofs thus proportioned have been lately exposed, without injury, to one of the heaviest cyclones ever known in South India, a storm which tore up trees by the root, and caused very serious damage to buildings of various kinds.

Robert E. Johnston, of Liverpool, M. I. C. E.—The question of wind pressure does not appear to have received the amount of attention from British engineers that the importance of the subject demands; this, no doubt, is due in a great measure to the fact that the wind in this country does not attain to anything like the force it does in America, but the destruction of the Tay Bridge has given a prominence to the subject it never had before, and has proved that even in this country the effects of the wind cannot, with impunity, be neglected. The extent of our information of a reliable character with regard to the pressure of the wind and the force it exerts against the complicated forms which the varied structures erected by engineers assume, is of a very limited nature, and it is desirable to determine by actual observation the pressure exerted by the wind at various velocities on flat, inclined, hollow and round surfaces. Investigation should also be made as to the action of the wind on close latticing, and the reduction that takes place in the pressure on the rear surface when two flat surfaces are immediately behind eath other, and at varying distances apart. Information of this nature would enable the floor and over-head bracing of open web girders to be determined with more precision than at present. It is usual to allow of a reduction of one-half as between the pressure on

the windward and leeward girders in this country, and, I think, a similar practice is followed in America, but there appears to be no reliable ground for this assumption.

I think the pressures Mr. Smith instances as having destroyed buildings must have been greater than he gives them, as I presume they were deducted from the moment of stability of those structures, which would not take into account the reduction in velocity from friction in passing over the ground. In my own practice, when determining the strains on roofs and other structures affected by the wind, I take the wind as exerting, in a horizontal direction, a pressure of 30 pounds per square foot, and I think this agrees with that adopted generally by English engineers.

In my opinion it must be admitted that the ground plan of a metallic skeleton pier, which is a parallelogram, is the one best suited to resist the strains, both vertical and lateral, to which it is exposed, and that the columns should be placed at its four corners in preference to the complicated grouping of the columns, as illustrated by the Crumlin and Fribourg Viaducts. The question of whether the base of the pier should be spread out so as to have no tension strains at that point appears to me to be one largely governed by the question of cost and local circumstances, for if a pier is to be placed in a navigable channel, or in deep water, it is desirable to keep down the dimensions of the coffer-dam or caisson to the lowest possible limit.

Under these conditions it would appear to be sound engineering to reduce the base of the pier and resist the tension of the iron portion by anchoring it down to the mason work, but when the masonry foundations can be provided without undue expense, the base should be enlarged so as to be stable without holding down bolts, in the manner adopted by Mr. Smith in his remarkable bridge over the Kentucky River, and almost to the same extent by M. Nordling, in his Viaducts on the Commentry & Gannat Railway, in France, where, at La Double, and other bridges erected by him at the same period, the total tension on the holding down bolts is reduced to about 20 tons by the spreading out of the base in a lateral direction. The piers of the Kentucky Bridge are mounted on rollers both in the longitudinal and transverse direction, and it would be interesting to know to what extent movement has taken place from expansion and contraction at the foot of the piers.

The highest pressures registered in Great Britain are those recorded

by Mr. Hartnup, at the Observatory at Bidston, and through the courtesy of that gentleman I am able to give the following particulars of the great storms of wind that have passed over that place between the years 1868–79:

Date of Observation.	Greatest Velocity in Miles between any hour and the next hour following.	Greatest Pressure in Pounds on the Square Foot.	Direction of Wind.
27th December, 1868	92	80	W.
13th October, 1870	82	65	W. S. W.
9th March, 1871	79	90	W.
27th September, 1875	81	70	W. S. W.
23d November, 1877	80	64	W.
28th December, 1879	59	38	S. W.

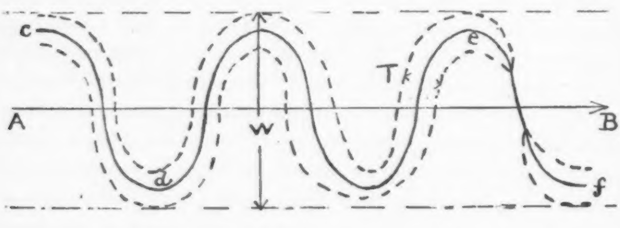
The velocities were registered by Robinson's anemometer, and the pressures by Osler's anemometer. For the last 30 years I have resided within a couple of miles of the observatory and am well acquainted with its surroundings, and having regard to the high pressures recorded, I think they can only be regarded as having been of momentary duration and of an entirely local character, for if the storms that gave rise to them had partaken of the cyclonic character of those enumerated by Mr. Smith, the houses in the neighborhood of the observatory must inevitably have been blown down, and no such a calamity has occurred in the district during the period I have mentioned, which is the strongest testimony that can be adduced in support of my opinion. If further evidence were required, I might state that I have been connected with a railway, that has its terminus about three miles from Bidston, for the last 25 years, and during this period no cars or freight trucks have been blown over, and as these have a stability of from 30 to 40 pounds per square foot, they cannot have been exposed to gales of wind with pressures ranging as high as 90 pounds per square foot. It may be of interest for the Society to know that Bidston, where the greatest wind pressures have been registered in this country is three miles from Liverpool on the Cheshire side of the Mersey and two miles from the Irish

Sea, standing on an eminence about 200 feet above sea level, being at the same time exposed to the full force of the wind from all quarters of the compass.

G. Bouscaren, M. A. S. C. E.—I wish to preface these remarks with thanks to Mr. Smith for his very valuable contribution on a most interesting subject, in regard to which so little is known by the best informed.

The author has, I believe, stated correctly in his paper what American engineers' practice in regard to wind strain on bridges has been in the past ten years. The question to elucidate in this discussion is, whether this practice is sufficiently safe, when considered in the light of present knowledge on the subject of wind power, and in the face of changes now being introduced in the rolling stock and machinery of American railways.

First.—It seems that the information given, concerning tornados in particular does not warrant the positive assumption, that their width of area of maximum effort is less than the ordinary limits of bridge spans. From my own observations, the general movements of a tornado seem to be of four kinds.



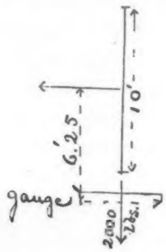
1st. The gyratory motion on its axis. 2d. The advance onward in the general direction, *A B*. 3d. An irregular oscillation to the right and left of the general direction, causing the axis of the vortex to describe a sinuous track, *c*, *d*, *e*, *f*, which, as stated by Mr. Smith, gives an apparent breadth of swath, *W*, much larger than the width, *T*, of the actual track.

4th. An irregular vertical oscillation, causing the width, *T*, which measures the diameter of the vortex to vary constantly, and to such extent as often to disappear altogether, the tornado jumping up, as it were, for a time, and leaving unharmed for a certain distance directly in its course, objects of comparatively little resistance. If I am correct,

the small observed width of the St. Charles tornado may be very reasonably explained by the supposition that the cone was well up in its vertical course when it struck the bridge. The margin between the pressure assumed in our calculations, and the highest pressure on record deduced from actual failures of structures, seems sufficient, but there is, nevertheless, reasonable room for doubt, which can only be dispelled by numerous and well directed observations.

The knowledge of the relations which the pressure of the wind bears to its velocity, and to the shape of surfaces exposed are important factors in the problem, but the great desiderata are accurate measurements giving the greatest effort actually exerted by the wind on the largest area.

Second. Mr. Smith makes the pressure per square foot necessary to upset a fully loaded freight train 56½ pounds, this can scarcely apply to prevailing railway practice at the present time. Box cars being now made to carry twenty tons and over, it would not be safe to reckon on a train load of less than one ton per lineal foot behind the engine. This



with an exposed surface of ten feet, and a height of 6¼ feet from centre of pressure to top of rail, which are maximum figures, gives for the upsetting pressure per square foot, 80 pounds for five feet gauge, and 75 pounds for 4′ 9″ gauge. Now, it is true, that a train would not run under a wind approximating such a force, for the tractive power of the locomotive could not overcome the train resistance with the additional friction of the wheel flanges against the rails, but the sudden apparition of a tornado, surprising a train moving or standing on a bridge or viaduct, makes it quite *possible* that the structure be subjected, with a train on it, to a pressure approaching 75 pounds per square foot, should the wind reach such a pressure. At all events, one conclusion is very clear

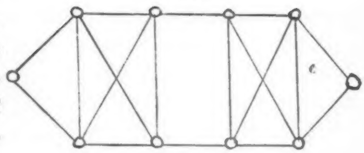
to me. We *know* positively that empty cars may be upset by the wind; this eventuality should be met with sufficient provision to resist it, in the design of bridges and viaducts.

Third.—It is undeniable that the rule of *no tension* in the columns of piers, with a good anchorage as an additional safeguard, is all that could be desired with reference to safety; but I do not think that it should be made absolute; in extreme cases it might involve a large additional outlay for masonry. With wrought iron columns well spliced, I do not see the objection of utilizing the masonry (with a large additional excess of weight) to resist the tension due to wind storms of exceptional force. Where cost can be reduced without sacrificing safety, it is not only legitimate but imperative to good engineering to do so.

Fourth.—In regard to the maximum strain per square inch allowable under wind pressure, my practice has been 15 000 pounds in tensile members, and as given by Mr. Smith for other parts, being equivalent to an addition of fifty per cent. to the maximum strains under the normal load. I have now come to the conclusion that this is too much, and in the future will only allow 12 500 pounds per square inch in tension. My reason for doing this is that the strains arrived at by calculation suppose that the force of wind is applied gradually; the strains due to the same wind, if applied suddenly (as would happen to a more or less extent in the case of a tornado), might be doubled in value, and reach 30 000 pounds per square inch, which is beyond the limit of elasticity of ordinary bridge iron.

In conclusion, I beg leave to put in a few words in behalf of our European brothers. It is but proper to say that the ground plan of Skeleton pier *A* given by Mr. Smith, as representative of the European type, is far from being generally used on the Continent. I do not know of a single case in France. Of the six French viaducts described by M. Nordling in his work, three are of the type *B* and three of the type *C*, of which is also the viaduct at Fribourg, in Switzerland, built by a French firm.

A



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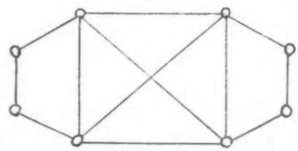

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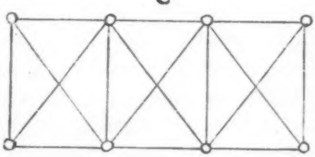
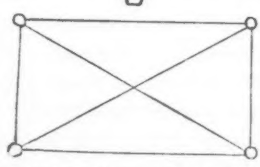


D



It is true that the piers of those structures are of cast-iron, but it must be remembered that they were all designed prior to 1864, at which time the use of cast-iron in columns was universal. This practice is now abandoned, at least by French engineers.

The bridge over the Douro, erected in 1877, by M. G. Eiffel, and the Garabit viaduct now being built by the same engineer for the Marvejols and Neussargues Railway are good examples of the present practice.

The piers in both structures are of the type *D;* the columns are of wrought-iron ; they are rectangular in section ; closed in the Douro plan, and open, with lattice work on one side, in the Garabit plan.

The practice, in regard to wind strain, is to assume a pressure of wind of 35½ pounds per square foot on the structure with a train on, 64 pounds per square foot on the structure without a train on, and take the highest results. The maximum normal strain allowed, which is about 8 500 pounds per square inch in tension, includes the strain due to the wind, which is often a considerable portion thereof. In the Garabit viaduct, for instance, the strain per square inch on different members of the 540 feet arch divides very approximately as follows :

	Strains per Square Inch in Pounds.			
	Due to Dead Weight.	Due to Rolling Load.	Due to Wind.	Total.
On the chords of 540′ arch.	2 833	2 833	2 833	8 500
On the web " "	1 400	1 400	4 200	7 000

Willard S. Pope, M. A. S. C. E.—The paper by Mr. C. Shaler Smith is a valuable addition to the stock of information on effects of wind. The provisions as to construction of bridges which his specifications call for, accord, I think, with the best practice of American bridge engineers of the present day. A structure designed and fairly built in accordance therewith will, doubtless, be safe as against the *lateral* pressure developed by any wind. But there is also a *vertical* tendency to be provided for. The effects of a whirlwind may be not only lateral, but also, under certain conditions, upward. Every one has noticed how the little eddies pick up dust and straws in the streets. I have seen on the plains of Wyoming and Colorado columns of dust, doubtless 200 feet high, with distinct and sharply defined outlines moving majestically over the ground, held within the vortex of a whirlwind. Ocean navigators tell of vast masses of water, weighing hundreds of tons, thus lifted up into the air, like great vertical pillars.

The mere upward spiral whirl of the disturbed air is probably insufficient to develop this tremendous vertical energy. It may, perhaps, be supposed that the central core of the tornado is a vacuum, more or less complete, formed and sustained, possibly, by the centrifugal power of the gyrating particles of air; and as the open base of this cylindrical, vertical, vacuous column trails along the surface of the earth, movable matter, exposed to its influence, rushes upward into it with a velocity and momentum due to atmospheric pressure as well as to direct force of wind. Roofs of buildings are frequently lifted off, although the only exposure to actual upward wind pressure is at the projecting eaves and cornices. But suppose this vacuous column passing over the building; with what explosive force the confined air within would shoot the roof upward, and the walls outward.

The existence of this vacuum seems essential to explain some of the

phenomena connected with tornadoes; and if such vacuum really occurs, it must develop a tremendous force vertically, just as the wind itself develops a force horizontally.

That such upward force may destroy a bridge I have some evidence. About 1858 a railroad, of which I was engineer, lost a bridge by wind. It was a wooden, through Howe truss, in two spans of about 100 feet each, with the chords framed together over the central pier. The bridge was not roofed or covered. The location was such that the structure was well sheltered by timber, and from the underside of the bridge to the surface of water in the stream was only about 15 feet.

The direction of the bridge line was east and west, and the storm passed from north to south.

When I reached the scene of the disaster, a few hours after it had happened, I found the spans in the river, badly wrecked, of course, but still continuous, and in tolerably fair line. The west end laid at the base of the west abutment, the central part just cleared the southerly end of the pier, while the east end laid probably about 20 feet southerly from the east abutment. The bolsters underneath the ends of the lower chords had been, when built, shouldered about two inches into transverse wall-plates of oak. These wall-plates (which had not been bolted to the masonry) were still lying in their places on both the east abutment and the pier, while those of the west abutment had been dragged into the river. If the spans had been tipped over, or had been slid off the masonry, by the horizontal force of the wind, they would certainly have taken the wall-plates with them. On the east abutment there was no sign of a blow from falling timber. There happened to be a joint in the track rails on the bridge within three or four feet from the east end. The ends of the bank rails were still protruding out over the edge of the abutment, and the ties attached to them had evidently been raised vertically from their beds in the embankment.

If the east end of the bridge had been lifted up bodily, and thereupon been moved 25 feet or so southerly, turning on the west abutment as on a pivot, and then dropped into the river, all these appearances would have been accounted for. I could make no other hypothesis that would explain them.

To lift this bridge required an upward pressure of about 80 pounds per square foot of exposed surface. This is on the supposition that the floor system was bolted to the trusses, which was the fact.

Applying this to a modern iron bridge of, say, 160 feet span: the horizontal surface of the iron work only is, say, 9 square feet per lineal foot of bridge. An upward pressure of 80 pounds per square foot would amount to 720 pounds per lineal foot. And as the iron work will weigh, say, 1 100 pounds per lineal foot, the structure will be safe. But the flooring thereon (for a railroad bridge) exposes, say, 11 square feet of horizontal area per lineal foot. This, at 80 pounds per square foot, amounts to 880 pounds per lineal foot, to counterbalance which is only about 300 pounds of dead weight of flooring. The upward pressure, therefore, on the two combined will be, say, 1 600 pounds, while the total resisting weight will be but 1 400 pounds.

The moral is, do not bolt the flooring firmly to the trusses.

If this reasoning is correct for railroad bridges, much more does it apply to highway bridges, where the area of flooring is generally much greater proportionately.

Joseph M. Wilson, M. A. S. C. E.—Mr. C. Shaler Smith's interesting paper on Wind Pressure seems to pretty well cover the subject, so far as present information on the matter allows, and agrees essentially with my own practice, except that in taking wind pressure at 30 pounds per square foot on vertical bridge surface with that on train at 300 pounds per foot lineal. I have been in the habit of taking only one truss, and have usually not exceeded the ordinary limits of working stress, such as 10 000 pounds per square inch tension on wrought iron, &c., &c., and also have not taken the pressure on the train as a moving load. I fully agree with Mr. Smith as to the propriety of taking this as a moving load, and also as to increasing the limiting stress; but in doing so, I should, as he does, take twice the vertical surface of one truss, and should be inclined to make my pressure 40 pounds, instead of 30, in very exposed localities.

The addition of an initial stress to allow for strains produced in screwing up the rods is a good point.

In some of our exposed buildings we have taken 25 pounds per square foot of surface on one side with no increase in working stress over ordinary limits, but in later work have adopted 30 pounds, with an increase in allowable stress of about one-third.

I should be glad to see some more reliable observations made on the matter of wind pressures at a point where high velocities are attained, Mount Washington, for instance, the question of surfaces of different

forms being considered, such as concave, convex, and those given by I beams, channels, &c. Also the loss of effect caused by the wind passing through the openings of one truss on to another, &c.

While I agree, of course, with Mr. Smith that the American form of trestle is decidedly preferable to the European form, still I think his criticism on the latter is a little severe, as the column *a* must be prevented from collapsing in the direction *c d*, at least to a great extent, by the bracing connecting it with the two adjacent columns of the trestle, and forming a triangle in horizontal section if such bracing is made heavy enough.

A. Gottlieb, M. A. S. C. E.—The views of Mr. C. Shaler Smith, with reference to wind forces and wind strains, as stated in his paper, meet my own views in every respect.

I believe that a wind pressure of 30 pounds per square foot on the side of a train, as also on the whole surface of the windward and such portions of the leeward truss, as may not be moved by the train, is sufficient for the safety of structures.

I have no doubt that winds of greater force than this occur, but considering that they are very exceptional, and as Mr. Smith states, their paths are narrow when they occur, I do not think it necessary to provide against larger wind forces than 30 pounds per square foot, certainly not for spans over 200 feet.

One point, I believe, however, Mr. Smith has not treated as fully as its importance calls for.

Every one of us has seen structures and bridges erected, with parts proportioned correctly to resist specified live as well as dead loads, as far as the sections of the members were concerned, and still the structures were failures, owing to defective details in the connections. So it is also with the members intended to resist and to transmit wind strains, and I might say in still a higher degree.

Engineers generally pay too little attention to this most important fact, and the consequences are twisted struts, bent pins and distorted chords; although the sections of these members, intended in the first place to resist wind strains, may even have been excessive.

In connection with this I will state, that I think it advisable that the sections of bottom chords of spans over 200 feet should be increased to resist the horizontal strains from wind strains.

The usual practice is to provide for the strains arising from live and

dead loads only in bottom chords, disregarding entirely the additional tensile strain, which is brought to act on one line of chord from the lateral rods.

I find that with the usual live load of two locomotives followed by cars and a wind force of 30 pounds per square foot, the additional stress in one line of bottom chord produced by the lateral rods over and above the strains derived from live and dead load combined, is for—

Spans of 100 feet, about 25 per cent.
" 200 " " 40 "
" 400 " " 50 to 60 per cent.

Taking the working strain in members of wind bearings and their connections 50 per cent. larger than for the vertical loads, the increase of section required in bottom and chords would be—

For 100 feet spans, 16 per cent.		
" 200 " " 26 "		Over and above the section usually given.
" 400 " " 33 to 35 per cent,		

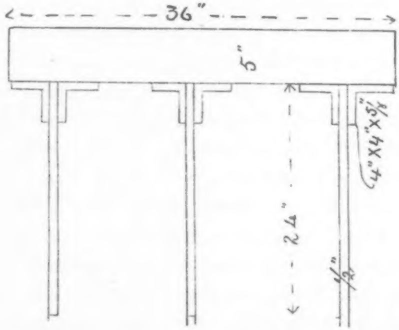
I believe that in all spans in which the section of bottom chords, required to resist wind strains, should exceed the section provided for live and dead loads by 25 per cent or more, this additional section should be given the chords, particularly as it could easily be done without increasing the dead weight of the structures materially.

The Chicago & Alton Railroad has lately adopted the rule to increase the section of every member of the bridge on account of wind strains, even floor beams, track stringers, top chords and posts. I believe this unnecessary, and rather detrimental to the structures, as it increases their dead weight, without adequate benefit to compensate for it.

D. J. WHITTEMORE, M. A. S. C. E.—This paper will have done excellent service aside from that due to its merit, if it will cause others to contribute such facts as to extent and ultimate concentrated force of tornadoes, as may come clearly and definitely within their knowledge. It appears to me that the same practice in designing structures to resist wind strains should prevail outside of the present known tornado belt that should be employed within it. Some eighteen or twenty years ago a tornado swept over and destroyed the village of Viroqua, Wisconsin, and in the supposed path of greatest effect carried wheat straw with sufficient velocity to cause it to penetrate through the one-half inch pine sideing of a destroyed dwelling house, and the piece of board with the

straws sticking through it is now in existence at the place mentioned. Who has the data from which to calculate the force required to cause this effect?

While our bridge experts generally conform to about the specifications set forth in Mr. Smith's paper there are many designers and builders of structures who do not—some in this country and some abroad. I would call members attention to a recent volume of the Proceedings of the Institution of Civil Engineers (Vol. LXI., p. 190), which contains a description of a viaduct built across the Clyde, at Glasgow, in 1878, which clearly shows the difference between English and American practice in regard to wind bracing. The viaduct carries four tracks and consists of five spans; each span has two riveted lattice trusses placed 50′ 0″ apart centre to centre, plate floor beams at the panel points, riveted stringers under each rail and a floor covered with Mallets buckle plates. The longest span is the part of the work I chiefly wish to refer to; its trusses are 199′ 0″ long, twenty feet in depth, and the floor beams apparently rest on and are riveted to their bottom chords. The panel length is 10′ 0″. Both top and bottom chords are built up of plates and angles as sketched, and the iron in them is strained to four tons (8 930) per square inch by the calculated loads both in compression and tension.



The description of the top lateral bracing is as follows (p. 197): "The tops of the girders over the river are connected by cross bracing 20′ 0″ apart. These braces are light lattice girders having the upper boom fish bellied. They assist to stiffen the viaduct, but the object for which they were erected was to support smoke boards, which were

erected, at the instance of the Clyde Trustees, to prevent sparks flying on to the shipping below."

From this we would infer that the writer of the paper thought that this span did not require any top lateral system as he states that the cross bracing was put in to carry smoke boards. Without lateral bracing the top chord would be a column of over 60 diameters strained to nearly 9 000 pounds per square inch, and the stability of the trusses would chiefly depend on their connection with the floor beams which are only 16¾" deep at the ends where they are riveted to the bottom chord.

It is impossible to estimate the stiffness which this arrangement gives to the structure without seeing the exact detail of the joint, and this is not given in this paper. But there can be little doubt that strong top lateral and portal bracing would greatly increase the strength and durability of the bridge.

A. S. C. Wurtele, M. A. S. C. E.—I have read with interest the advance copy of paper by C. Shaler Smith, on Wind Pressure on Bridges, and consider it a full statement of a subject which has not received sufficient attention; my experience, from examination of plans as consulting engineer, being that the additional strain in chord members, due to wind pressure, is quite neglected.

I know of no definite results as to the effect of such pressure on open lattice work, but do not think it equal to the actual surface, for whilst lowering a 60 lb. span lately, during a high wind, I found the guy lines only slightly strained, the support being only two long jacks at each end.

From the double bearing the calculation of swing strains is indefinite and comes under the class of imperfectly fixed trusses, so that a complete reduction would give rather a fanciful result; but it is best to assume the worst condition of one side only fixed.

I prefer to use the same specified strain as for a load with a lesser wind intensity, so as to get sections of combined strain at one operation —at 20 pounds pressure would equal 50 pounds before elastic limit would be reached under load strains.

Chas. A. Smith, M. A. S. C. E.—I have observed when standing upon high bridges and other exposed situations, that very frequently the wind came in regularly recurring periods or pulsations. In three instances at St. Charles, and one at Plattsmouth, the interval between pulsation was about one second. These were not the irregular gusts which, of course, occurred, but continued for hours. It is possible that such

action of the wind could produce oscillations in the structure itself, if the periods agreed with that of lateral vibration, or if one was a multiple of the other.

Concerning the values used it would appear that the pressure recommended by Theodore G. Ellis, M. A. S. C. E.,* and adopted by F. Slataper, M. A. S. C. E., would be preferable to members of the community who are not manufacturers of bridges, and also would give a better margin for the effects of vibration and that unknown element, viz.; the lateral force required to keep a long compression chord in line which is theoretically nothing if the chord is in line, and which increases with the allowable deviation. If the 30 lbs. allowed be enough for the wind, does it include this effect or not, and will not the higher value of 40 lbs. give a structure with less lateral vibration? The amount of metal in question is not very great, and the life of the bridge must be increased thereby.

Robert Fletcher, Asso. A. S. C. E.—Confessedly our knowledge concerning this subject is imperfect, and we have scarcely any data or formulæ upon which we can confidently rely. The mistake of attempting to deduce conclusions as to the actual pressure of strong winds upon large stationary objects from the velocity of rotation of the arms of an anemometer, is admitted. The probability,—we may almost say certainty,—that the intensity of pressure on the exposed surfaces of high structures increases from below upwards, throwing the centre of pressure above the centre of figure, has been referred to by J. B. Eads, M. A. S. C. E., in a former discussion,† and also by J. B. Francis, M. A. S. C. E., in his valuable report on the stability of the chimney of the Lawrence Manufacturing Company.‡ If there is any law for this increase, it must be dependent upon special and perhaps obscure conditions in each case, and it would not be an easy task to determine it. The influence of the size and form of the resisting object and of the partial vacuum shown by the eddy have also been noticed in the previous discussion. The latter condition is one of great importance.

Weisbach shows that for the pressure of moving water against an obstruction we may write, theoretically,

$$P' = Fhj + C' \frac{v^2}{2g} Fj,$$

*See Transactions, A. S. C. E., Vol. IV, page 132, May 1875.

† Transactions, Vol. IX, p. 395.

‡ See Engineering News, Aug. 28, 1880.

(Mechanics of Engineering, Vol. 1, p. 1030) in which F is the surface pressed, h the hydrostatic head, j the heaviness of the water,—therefore Fhj is the hydrostatic pressure,—v the velocity of impulse, C' an empirical co-efficient dependent upon the shape of the surface and, therefore, the second term expresses the pressure due to impact. Similarly against the back surface, where the eddy occurs, the expression is $P'' = Fhj - C'' \frac{v^2}{2g} Fj$, in which C'' is an empirical co-efficient for this case. Hence for the resulting impulse we have

$$P' - P'' = P = (C' + C'') \frac{v^2}{2g} Fj.$$

He then says: "This general formula for the impulse and resistance of an unlimited stream is also applicable to the impulse of wind and to the resistance of air. Here, however, besides the difference of the ærodynamic pressures upon the front and rear surfaces, a difference in the ærostatic pressure also exists, which is due to the fact that the air at the front surface has a greater heaviness (j) in consequence of its greater tension, than that at the rear surface." Another remark is of importance in this connection: " * * * a certain quantity of air or water attaches itself to the body, the influence of which is shown by the variable motion of the body, which, *e. g.*, is very evident in the oscillations of a pendulum. The quantity of air or water which attaches itself to a sphere is 0.6 the volume of the sphere. For a prismatic body, moving in the direction of its axis, the ratio of these volumes is

$$0.13 + 0.705 \frac{\sqrt{F}}{e}$$

in which e denotes the length and F the cross-section of the body. This ratio, which was first determined by Du Buat, has been fully confirmed by the later experiments of Bessel, Sabine and Bailly." For the impulse of air and water upon a plane surface, the experiments of Du Buat and also of Thibault give $C' + C''$, or, more simply, $C = 1.86$, about two-thirds of the action being upon the front, and about one-third upon the rear surface. This was for a uniform velocity of impact. Ashbel Welsh, M.A.S.C.E., in commenting* upon the experiments of Beaufoy on the resistance to prismatic bodies moving through water, infers that in air this co-efficient would be greater than Beaufoy's value which was 2.3. But the results obtained by Du Buat, Duchemin, Newton, Borda,

* Transactions, Vol. IX, p. 392.

Hutton and others, as cited by Weisbach, for prisms and spheres in water and air, at moderate and high velocities, show no values as great as 1.5.

After all that has been done we are still left in doubt. Previous experiments generally were made on too small a scale, or under conditions otherwise not according with those with which the engineer has to deal. Hutton has given us an empirical formula which involves the inclination of the exposed (plane) surface to the direction of the wind. It is especially applicable to roofs,* but requires us to assume a certain arbitrary intensity of horizontal wind pressure.

It is evident, then, that while theories and formulas based upon previous experiments and observations by distinguished professional men are valuable, nevertheless, we must put these in the background, and call for actual facts and data derived from careful and close observation, under conditions, usual or possible, of our practice at the present day. The paper of Mr. C. Shaler Smith is a valuable contribution, and goes far towards meeting this demand. We need a great deal more of the same kind of information, and then a thorough comparison and digest of the results embodied in the form of practical rules. But, of course, in applying results of such observations to a particular case, great care must be taken to see that the conditions correspond as nearly as possible to those occurring in connection with the results to be made use of.

I have had in view for several years past the making of systematic experiments on wind pressure, but have not yet found either time or means or suitable facilities at hand. And, indeed, no one person can generally command the means and access to all the localities necessary for the proper prosecution of such a work. Necessarily, there must be co-operation and separate experiments by a number of engineers, according to some carefully prearranged plan. I would offer the following general suggestions for such a series, leaving detail and additions to be suggested by more experienced members :

1st. A sufficient number of self-registering dynamometers should be provided for use at different points for direct and simultaneous measurement of the pressures, somewhat after the method pursued by Thomas Stevenson in his measurements of the force of the waves,† but on a

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*See Greene's "Graphical Analysis of Roof Trusses," Chapter VI.

† See Stevenson's "Design and Construction of Harbors," Chapter IV.

much larger scale. Different series would have to be conducted for different classes of structures, as follows :

2d. Surfaces high, broad, and substantially plane, as the faces of large, high buildings, observations so as to show the variation of pressure from below upwards, and both on the windward and leeward faces. The mechanism of self-registering need not be made to operate for a long time, if the instruments could be set at the beginning of a storm, and watched. In this connection, I think it would be important to observe the amount of partial vacuum at one or more points of the leeward face, which might be done with aneroid barometers.

3d. Surfaces high, narrow, and substantially unbroken, as high masonry towers, chimneys, high piers, etc., observations similar to those just described. In these cases the eddy or partial vacuum on leeward side is probably more, relatively, than in case of large buildings. The great importance of this effect may be inferred from the remarks of Robert Briggs,* M. A. S. C. E., in the former discussion, and by considering that a variation of half an inch only in the barometric pressure before and behind, means a preponderance of ærostatic pressure amounting to about ¼ pound per square inch = to 36 pounds per square foot, which is in excess of pressure due to the impact alone from a very high wind. Mr. C. Shaler Smith bears testimony to the influence of this partial vacuum in stating that nearly all the buildings he has examined in the tracks of western tornadoes appear to have blown outwards. Many will remember the curious illustration of this principle shown at the Philadelphia Exposition,† where a ball of considerable weight was sustained on the *lower* edge of a jet of air from an air-compressor, said jet having an inclination of 60 degrees to the horizontal. The partial vacuum on the leeward side of the ball, caused by the jet above the ball and a small part which passed underneath, made a preponderance of ærostatic pressure acting upwards, and the resultant of this and of the force of impact was sufficient to counteract the weight.

4th. Similar observations on framed structures, as trusses and trussed towers and piers, both near the surface of the earth, and in specially high and exposed situations. In these cases it would be interesting and important to learn more certainly than we now know, the shielding effect of one part of the structure upon those parts not receiving the first impact of

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* Transactions, Vol. IX, p. 398.

† See *Scientific American* Supplement, Vol. II. p. 736.

the wind. Also the amount and influence of the contraction of the stream of moving air as it passes between the parts of the truss. Probably in many structures, or behind certain parts of large frames and trusses, a very considerable degree of exhaustion of air would be observed.

5th. In some cases, during brief intervals, with enough observers working together, aneroid barometers simply might suffice to measure approximately, if not, indeed, as accurately as some other apparatus, the pressures on windward and leeward faces of structures or parts of structures.

Such a series of experiments would show the influence of height above the earth's surface, the form and size of the surface or combination of surfaces, the effect of inclination to direction of the wind, and of other conditions which would be suggested.

Manifestly, it would be necessary to have simultaneous observations of velocity, and some means of noticing the exact direction of the wind, so far as practicable.

O. Chanute, M. A. S. C. E.—I have heard some members ask, why if he has observed a wind pressure of 93 pounds per square foot, Mr. Smith still thinks it safe to proportion his structures for 30 pounds per square foot. They urge that in the course of years, at some time or another, those extreme pressures will be realized.

I apprehend that they have not clearly understood Mr. Smith's argument. Not only, as he states, are those extreme pressures limited to very narrow belts—less than the length of ordinary spans ; not only would a train be blown over by a far less pressure, but it is most unlikely that a train would venture across a bridge during the prevalence of such a tornado.

In that case the exposed area, instead of being 21⅓ square feet per foot lineal, for train and trusses, would be 11⅓ square feet, for the latter alone. Upon this a pressure of 90 pounds per square foot would amount to 1020 pounds per lineal foot, instead of the 640 pounds per lineal foot (estimated at 30 pounds to the square foot), upon the bridge and train; so that if the latter strains the wind bracing 15,000 pounds to the square inch, the tornado will strain it 24,000 pounds to the square inch; which, as Mr. Smith says, is still within the limit of elasticity.

There are one or two additional considerations which Mr. Smith might have added to his very interesting paper, to sustain his position

that our current American practice in regard to wind strains on bridges need not be essentially modified.

The following occur to me:

1. All calculations of wind pressures are assumed to be at right angles to the axis of the bridge. The resultant of this pressure is, however, I believe, in proportion to the sine of the angle, which the wind makes with the surface against which it is blowing, so that inasmuch as tornadoes (which alone, as Mr. Smith has pointed out, can exceed the pressures assumed) are not likely to strike the bridge very often at right angles, the probabilities of these extreme strains being exerted are still further reduced,

2. Not only is this pressure of 30 pounds per square foot, calculated, as stated by Mr. Smith, upon "twice the vertical surface of one truss," but the usual modes of estimating this truss surface, contain a still further element of safety.

Many engineers calculate this pressure upon the *developed* surface of the members of the truss, measuring around all the angles and projections; while Mr. Smith himself, as I understand from a private letter, estimates the exposed surface of *one truss*, by adding to the geometrical surface as shown on the drawings for upper chords and posts, one and a half times the surface of the ties, and twice the surface of the lower chord.

It is somewhat curious that nobody who has taken part in this discussion has yet called attention to the influence which the *form* of the bridge members has upon the resulting pressure. We all know that a convex surface presents *less* and a concave surface *more* resistance than a flat plane to the wind. I dare say that some of us have ascertained this fact experimentally with an umbrella, but I do not know that this knowledge has ever been applied to the calculation of the wind strains on bridges; or that any experiments have been made to determine the comparative resistance of various forms of bridge members.

I believe that Sir George Cayley found that a sphere offered a resistance of but 42 per cent. that of a flat plane taken through its mid section, and as the developed surface of the semi-sphere is equal to two great circles, the *average* of the pressure in that case per square foot is but 21 per cent. of what it would be upon a flat surface of equal area.

Again, an oblate spheroid of 3 diameters offers but 21 per cent. of the resistance of a plane through its mid section, so that the *average*

pressure per square foot, in that case, is but 6 or 7 per cent. of that of a plane of equal area.

The umbrella, which I mentioned a while ago, offers, I believe, a resistance of but 76 per cent. that of the plane drawn through its base, if turned with its convex side to the wind, while the concave side offers a resistance of 1.9 times that of the plane.

If we apply this knowledge to bridge members, we see at once that convex objects, like round ties and round closed columns, will offer less resistance, while trough-shaped posts and top chords will give more resistance than the plane surfaces shown upon the drawings. It is probably the general consideration of these facts which has led engineers to calculate the resistance upon a surface greater than that shown on the drawings, but it might be well to try a few experiments upon the actual wind resistance of various forms of bridge members, in order to know more accurately what is a safe allowance to make.

AMERICAN SOCIETY OF CIVIL ENGINEERS.
INSTITUTED 1852.

TRANSACTIONS.

NOTE.—This Society is not responsible, as a body, for the facts and opinions advanced in any of its publications.

CCXXI.

(Vol. X.—May, 1881.)

AN EXAMINATION INTO THE METHOD OF DETERMINING WIND PRESSURES.

By F. COLLINGWOOD, Member A. S. C. E.

READ APRIL 6TH, 1881.

WITH DISCUSSION

By WILLIAM E. WORTHEN, Member A. S. C. E.

About two years ago, the writer prepared some notes on the subject of wind pressures, which, upon consideration, he decided not to present to the Society. Subsequent events have led to a modification of some of the views then held, and with these modifications they are now presented. On account of the inherent difficulties surrounding the subject, engineers have seemed to acquiesce in an empirical solution of the problem, and to decide that nothing better could be done. This is not in accordance with the spirit of investigation now extant; neither does it tend to economy or safety in construction; and it would seem that the time had come for a more thorough treatment of the matter.

In a paper by John Smeaton on the "Construction and Effect of Windmill Sails," read before the Royal Society May 31 and June 14, 1759, he gives a table of wind pressures, which have been quoted as authority in nearly all the hand and text books published since, and which is adopted by the United States Signal Service. He prefaces it thus:

"The following table, which was communicated to me by my friend Mr. Rouse, and which appears to have been constructed with great care, from a considerable number of facts and experiments, and which, having relation to the subject of this article, I here insert as he sent it to me; but at the same time must observe, that the evidence for those numbers where the velocity of the wind exceeds 50 miles in an hour, do not seem of equal authority with those of 50 miles and under. It is also to be observed that the numbers in column 3 (giving perpendicular force on one foot area in pounds avordupois) are calculated according to the square of the velocity of the wind, which, in moderate velocities, from what has been before observed will hold very nearly." (In a foot-note elsewhere he explains that certain experiments by Mr. Rouse were made with windmills by a special machine constructed so as to move the wheel (or sails), as a whole, through the air, and noting the weight lifted by the wheel with varying velocities of movement. The table is, therefore, apparently based on resistance by the fluid to motion through it, and *not* on impulse or presssure from the air. Whether these experiments are the ones on which the table is based is not stated, but may be inferred, although from what follows, it would seem that the table was computed largely).

Referring to Weisbach's Mechanics (Vol. I., § 390, Ed. 1848) the formula for the pressure tending to move a prismoidal body by the force of an unlimited stream of uniform velocity in which it is immersed is

$$P = (f_1 + f_2)\ \frac{v^2}{2g}\ d.$$

Where P = pressure in pounds per unit of cross-section.

v = velocity of stream in feet per second.

f_1 = co-efficient of pressure at up stream end.

f_2 = " " " down " "

d = weight of a unit of volume of the water.

He says that where the length parallel to the axis of the stream is zero (corresponding to a thin plate), Du Buat & Duchemin found that the sum of the co-efficients

$$(f_1 + f_2) \text{ will} = 1\tfrac{86}{100} \text{ or } P = 1.86 \frac{v^2}{64.4} d.$$

For a length of $\dfrac{l}{\sqrt{\text{section}}} = 1$, the co-efficient $= 1.47$.

" " " $= 2$, " $= 1.35$.

" " " $= 3$, " $= 1.33$, &c.

And as the ratio increases the coefficient continues to diminish.

Taking the case where it equals 1.86 as being nearest to the windmill sails, and substituting 0.07925 pounds as the weight of a cubic foot of air, we have where P = pressure in lbs. avordupois and V = velocity in miles per hour, $P = 0.00492\ V^2$. (1).

This equation gives results the same as those given by Rouse in the Smeaton tables.

Dubuat found that where the ratio $\dfrac{l}{\sqrt{\text{section}}} = o$ (or the case of a thin plate moved *against* water) the co-efficient became 1.25, or the pressure is 33 per cent. less than where the water moved against the plate. This is given in article "Resistance" in Brand & Coxe Cyclopedia as 1.43. In the same article a cylinder, with hemispherical end facing the stream is said to offer only half the resistance of the same with a plain end. Also that while the impulse of the stream against a plate 1ft. square gave the co-efficient as $1\frac{9}{10}$, a plate with one-tenth of a square foot of surface gave it as only $1\frac{4}{10}$, or nearly 20 per cent. less. Hutton's theoretical deductions, as stated by Weisbach, for a sphere moving against air give the co-efficient as increasing from 0.59 for a velocity of 1 metre to 1.10 for a velocity of 600 metres. In Hutton's mathematics the resistance to a spherical projectile at high velocities is given as 0.43 of that to a plain surface.

Of a hemisphere convex side forward 40 per cent.

" " plane " " 101 " &c.

In Appleton's "Cyclopedia of Applied Mechanics," 1880, is a table of pressures calculated by Mr. Alfred R. Wolff in which account is taken of the difference in density of the air at various temperatures. At a mean it differs but slightly from Smeaton's table, and would seem to be based on the same formula.

Prof. Cleveland Abbe in the article "Wind," in Appleton's American Cyclopedia, Vol. 16, writes as follows:

"Maxwell (see Proceedings of the Mathematical Society, 1870) has given theoretical formulas and curves showing the movements of par-

ticles of an incompressible fluid streaming past a moving obstacle; while Hagen (Berlin, 1872), has experimentally investigated these motions. Thiesen (Wild's Repertorium, 1875) has made a careful study of the experiments of Hagen and Dohrandt, and established the rule that the pressure of the wind against an inclined rectangular plate is really very nearly proportional to the square of the velocity, and the cosine of the angle of incidence of the wind, while the absolute value of the normal pressure is as given by Hagen's observations. The latter physicist (Berlin, 1874) has embodied the results of very careful observations at moderate velocities in a formula which converted into English measures is as follows :

$$P = (0.0028934 + 0.0001403\, p)\, SV^2 \qquad (2).$$

By introducing the term (p) Hagen has expressed the fact that the pressure depends to a considerable extent on the *shape* as well as the surface of the resisting body."

In this formula $P =$ the total pressure in pounds avordupois ; $p =$ the outline or perimeter of the exposed surface in feet ; $V =$ velocity in miles per hour ; $S =$ the area in square feet. The formula applies to plain surfaces (of no considerable depth) placed normal to the incident wind, and with the density of the air corresponding to a barometric height of 29.84 inches, and a temperature of 59° Fah.

The results from equations (1) and (2), together with Smeaton's and those of Hutton, are given in the table on the following page.

All are taken for one square foot of surface. It will be seen that Hagen's are about one-third less than Smeaton's, and Hutton's about midway between the two.

The diversity of views among practical men in determining the measure of surface acted upon, the probable wind to be resisted, and how the wind acts, are well shown by the evidence given at the inquest on the Tay Bridge disaster. These differences are greater even than those in the tables.

The journal *London Engineering*, January 2, 1880, in discussing the accident, takes the lee girder as exposing one-half as much surface to the wind as the windward girder, and the whole vertical surface of the train also as acted upon ; and deduces according to different supposed modes of failures pressures from $23\frac{1}{2}$ to $35\frac{6}{10}$ pounds per square foot, expressing the belief, however, that the maximum pressure did not exceed $25\frac{1}{2}$ pounds.

Velocity of Wind.		Pressure in Pounds per Sq. Ft. Normal Plane Surface = P.						Names Given to Winds.		Classification by Smithsonian Institution.
				Hagen, Eq. (2)						
Feet per Second.	Miles per Hour = V.	Rouse, Eq. (1).	Hutton.	Circular Surface.	Square Surface.	Triang'lar Surface.	Smithsonian Institution.	Smeaton—Rouse.	Smithsonian Institution.	
0.	0	0.	....	....	....	....	0.	....	Calm	0
1.47	1	0.005	0.004	0.003	0.003	0.004	....	Hardly perceptible	....	....
2.93	2	0.02	0.016	0.014	0.014	0.014	0.02	Just perceptible.	Very light breeze	1
4.4	3	0.044	0.036	0.03	0.031	0.032	....			
5.87	4	0.079	0.065	0.054	0.055	0.057	0.08	Gentle, pleasant, wind.	Gentle breeze	2
7.33	5	0.123	0.101	0.085	0.086	0.088	....			
14.67	10	0.492	0.405	0.339	0.345	0.353	....	Pleasant, brisk, gale		
17.6	12	0.711	0.583	....	....	....	0.75		Fresh wind	3
22.	15	1.107	0.911	0.763	0.777	0.795	....			
29.34	20	1.968	1.62	1.356	1.382	1.413	....	Very brisk.		
36.67	25	3.075	2.531	2.119	2.159	2.208	3.		Strong wind	4
44.01	30	4.429	3.645	3.052	3.109	3.18	....	High winds.		
51.34	35	6.027	4.961	4.154	4.232	4.328	6.		High wind	5
58.68	40	7.873	6.48	5.425	5.527	5.653	....	Very high.		
66.01	45	9.963	8.201	6.866	6.996	7.155	10.		Gale	6
73.35	50	12.3	10.125	8.476	8.637	8.833	....	A storm or tempest	....	....
88.02	60	17.715	14.58	12.208	12.437	12.719	18.	A great storm	Strong gale	7
110.	75	27.65	22.781	....	....	....	....	....	Violent gale	8
117.36	80	31.49	25.92	21.701	22.11	22.613	....	A hurricane	....	....
132.03	90	39.852	32.805	....	....	....	....	....	Hurricane	9
146.70	100	49.2	40.5	33.908	34.546	35.332	....	A hurricane that tears up trees and carries buildings before it	Most violent hurricane	10

In subsequent numbers of *Engineering*, evidence is given as follows:

Mr. Allan Stewart placed the pressure at 75 pounds. Mr. B. Baker placed it at 15 pounds; he had experimented on glass in frames, and anything like 40 pounds per square foot would have destroyed the signal boxes. He had never seen a structure blown down which was capable of withstanding 20 pounds, and he looked upon that as the maximum pressure to be taken for structures of any kind. Fences capable of withstanding only 13 pounds had withstood storms in all situations.

He states that the French engineers assume $34\frac{6}{10}$ pounds as the greatest pressure a train can bear without overturning. One of their engineers had gone into an examination of chimneys, and had assumed an arbitrary figure of 55 pounds as the force they had to withstand, and this figure was now taken as the pressure against bridges, &c. The same has been taken by Rankine.

Mr. Baker said that in his own work he assumes 28 pounds, and varies the *factor* of *safety* with circumstances.

Dr. Pole and Mr. Stewart estimate the pressure required to overturn the bridge with no train upon it at $37\frac{4}{10}$ pounds, and with the train, at $34\frac{1}{3}$ pounds (also, $28\frac{1}{2}$ pounds as needed to overturn the lightest carriage in the open, and 33 pounds when containing 8 passengers and shielded by the bridge members).

Mr. Law places the same at $36\frac{3}{8}$ and $32\frac{3}{8}$, respectively, and that 40 pounds were needed to overturn the lightest carriage in the train.

Prof. Airy states that during the gale of December 8, 1872, the greatest velocity recorded was 57 miles per hour, and he thought the pressure had reached 50 pounds per foot. The maximum pressure recorded at Greenwich was 40 pounds and then the instrument gave way.

He had estimated the pressure in the valley of the Tay at 50 to 100 pounds. "The valley runs deep, and would make a channel for the wind." He thought 120 pounds ought to be provided for, but this would allow a large margin for safety.

He did not know whether such pressures ever obtained simultaneously over so large an area as the Tay Bridge. In a letter respecting the Forth Bridge, he had stated "that over very limited surfaces and for very limited times the wind pressure was sometimes known to reach 40 pounds in England, and in Scotland would probably amount to

more." The greatest that might be exerted over the whole extent of such a surface as the Tay Bridge was 10 pounds.

Mr. Gilke, previous to the accident, read a paper in which he took the total surface exposed at about two-thirds of the amount assumed by *London Engineering*, and the pressure needed to overtun a carriage at $39\frac{2}{10}$ pounds.

Prof. Stokes, of Cambridge University, at the inquest, said he thought 10 pounds per square foot, on a plain surface, as too low. The more rapid the *fluctuations* in the wind, the less their extent laterally. As far as experiment went the pressure was found to be nearly in proportion to the area of surface. He should like to see an exhaustive series of experiments made on large surfaces. There were difficulties in recording velocities of the wind for short spaces of time. There is a difficulty in connecting the velocities with the pressures. Hydrostatically measured, a pressure of one pound on the square foot gave a velocity of 20 miles per hour. Experiment had shown that about 20 per cent. must be added to what was called the standard pressure (that on the surface of water in a tube), in order to get the algebraic sum of pressures per foot on both sides of a plate.

Mr. Scott, Secretary of the Scottish Meteorological office, said: "The greatest velocity registered at Glasgow (during the storm which destroyed the bridge), for any continuous 60 minutes, was 71 miles; but for one five minutes interval, it was 96 miles, for one 3 minutes interval, 120 miles, and for three others, 110 miles per hour." The storm-gust at the Tay Bridge was probably 300 feet wide, and some gusts may be restricted to 30 or 40 feet width.

Passing to other known disasters from wind pressure, the destruction of the Arrah road bridge across the Sone canal in India, is worthy of mention, since the weights, friction, &c., have been accurately calculated (see *London Engineering*, June 25, 1880).

This was 99 feet long, 15 feet wide, and continuous over two spans of 45 feet each. It was destroyed by a very narrow belt of wind. There were no holding down bolts, and the structure was lifted and pushed bodily off the supports, with almost no injury to the masonry.

Taking the whole vertical surface of both trusses, and the wind horizontal, and assuming the friction at 50 per cent. of weight, the pressure on the supposition of the wind striking the uncompleted span first, would be $20\frac{8}{10}$ pounds. If moved bodily, it must have been $29\frac{6}{10}$

pounds. Calcutta was the nearest point where records were kept, and during the cyclone 50 pounds were registered.

Coming nearer home, Mr. Francis in his report on the chimneys of the Lawrence Manufacturing Company (published in *Engineering News*, Aug. 28, 1880), states the highest recorded pressure for 50 years past on the New England coast to be 50 pounds.

Mr. C. Shaler Smith, in his paper before this Society of December 15, 1880, states pressures in the Western States at from 18 to 93 pounds, but does not state how the results were arrived at.

Mr. W. Hartnell, in a discussion in *London Engineering* of January 2, 1880, states, respecting the gale in 1868, during which a momentary pressure of 80 pounds was recorded at the Bidston Observatory in Liverpool, that on the side of the observatory farthest from the sea, the pressure was only 38 pounds. He says further, that behind a flat plate a *partial vacuum* is formed by wind rushing past the edges. One per cent. of vacuum would give an increase of pressure per foot of 21 pounds. He says that narrow, flat bars, and open structures generally, offer a *maximum* resistance per foot of area. So that a statement of so much pressure means nothing, unless we know just how it is taken. He also says that a squall in still air will travel, say 20 miles per hour, and will blow at 30 miles for a short time, thus "exerting twice the pressure that might have been expected." In other words, that the maximum pressure is twice that due to the average velocity of the wind.

It is not necessary to go farther to illustrate the conflict of opinion which surrounds every branch of this subject.

There can be no doubt that much of the uncertainty arises from the use of imperfect apparatus, the *inertia* of the parts being entirely overlooked. On this point the article "Anemometer," in the *Cyclopedia Brittannica*, although written some years ago, is in a measure true now. It says: "If the currents of air were anything like uniform, it would be a comparatively simple matter to deduce the velocity from the pressure; but their variability is so great, that the relations between them become unworkably complex. We know from the elementary principles of dynamics that the pressure at any instant will vary as the square of the velocity. Obviously, therefore, the relative variations of the pressure will be *twice* as great as those of the velocity; and the *latter* are too great, as we find them to encourage us to double them artificially. It must be remembered also, that from the *inertia* of the indicating ap-

paratus, errors will in every case arise, and *these* also will be doubled if we take the pressure instead of the velocity indications. From all this it will appear that comparatively little importance is to be attached to the earlier, and to *all* statistical anemometry."

As an example of the apparatus used for recording pressures, may be instanced that at the Central Park, in New York, as described in the report for the year ending December 31, 1869. A storm occurred December 18, 1869, which is fully recorded in the same report. The apparatus consists of a hollow metal cylinder 2 feet high and 1 foot diameter, suspended with its axis vertical by a chain, and having a chain extending from its lower end, and connecting with a spring balance. As the wind blows the cylinder to one side, the spring is extended, and a pencil attached to the spring is moved up, and records its excursion on a sheet of paper traveled slowly past it by clock work. By testing the power required to extend the spring to various points, it would seem that a fair measure of the pressure might be obtained.

On examining the record of the storm mentioned, however, it is seen to be a series of quick strokes outward, and back again nearly to the zero line. So much is this the case, that the paper is shaded almost black near the zero line, becoming lighter as we pass from it, and with a few detached strokes of no appreciable duration at the extreme pressures. The *greatest* pressure recorded is 22 pounds, and that only twice, and at long intervals. Of 18 pounds pressure or over, there were four at intervals of 12½, 30 and 35 minutes respectively. At 15 pounds and over, about 10, with intervals of 2½ to 30 minutes. These are approximations, as it is difficult to measure from the lithograph.

It is evident that these records are vitiated by the *inertia* of the cylinder, just as when a pound weight is suddenly placed upon a spring balance, about 2 pounds will be momentarily indicated.

The instrument can in no case, however, be considered a scientific one, and there seems to be a similar difficulty with most of those employed.

Another source of error is well pointed out by Mr. Ashbel Welch, in his paper before the Society, of May 25, 1880.

If we take the fundamental formula $h = \frac{v^2}{2g}$ and calculate the pressures corresponding to air at the standard temperature and density, the results given will be very closely one-half only of those given by Rouse

and Smeaton. We know, however, that the actual pressures from the wind are greater. Could each particle of air give up all its motion, and get out of the way of those to follow without affecting the latter, the result would be rigorously true. There is, however, a piling up or condensation at the front or incident surface and a lateral flow—such, that the stream to be considered has virtually a larger section or base than the surface acted upon. In addition, there is the partial vacuum in the rear, noted by Mr. Hartnell, but first pointed out by some of the earlier experimenters on the subject.

These results, as we have seen, are modified not only by the shape of the incident surface, but also very largely by the length of the body acted upon in the direction of the wind current.

To sum up from the data given, we find the following sources of error, if we consider the maximum pressure of the wind, upon the exposed surface of a structure, as required by the engineer in proportioning its parts :

1st. Imperfect anemometers, vitiating results by the inertia of the moving parts, thus tending to make results too large.

2d. The difficulty of translating velocities into pressures, tending to make results too *small*, since the velocity recorded is an average over a period of time, and not that at the instant of greatest pressure.

3d. The data on record are not exact. To serve the purposes of science every circumstance as to size of parts, character and shape of surface, size of openings, height above ground, and location, must be noted.

4th. Almost all the records are based upon the assumption of a horizontal wind, whereas all experience shows that this is incorrect, and that the largest possible projection is liable to be the one acted upon.

In reference to height above ground, and direction of wind, the following memoranda are to the point.

In *Engineering News* of September 11, 1880, is an article referring to experiments with a specially devised anemometer, by Mr. S. Fraser, of the Scottish Meteorological Society. He says that wind currents are much modified by the ground over which they pass. For example, in front of the Royal Observatory in Edinburgh, their deflection from the horizontal was about 45°. On the level ground it was not over 15°, except in one case near Edinburgh, when it suddenly assumed nearly a *vertical* position. In the case already mentioned of the Arrah road

bridge, it would seem that there was a strong lifting, as well as a horizontal force, to have removed the bridge with so little damage to the masonry.

As to the effect of height there is in the monthly weather report of the U. S. Signal Service for September, 1880, the following quotation from a "Report on Results of Wind Observations, made with small cup and dial anemometers at different heights, by Thomas Stevenson, M. I., C. E., published in the Journal of the Scottish Meteorological Society.

"Although additional observations are much wanted at high levels, the results, so far as appears from the observations on winds varying from 2 miles an hour to 44 miles an hour, show:

1st. That spaces passed over in the same period of time by the wind increase with the height above the sea level, or above the surface of the ground.

2d. The curves traced out by those variations of velocity (from 15 feet to 50 feet above the surface of the ground, and possibly higher) coincide most nearly with parabolas having their vertices in a horizontal line 72 feet below the surface.

3d. Between 15 feet and the ground surface there is great disturbance of the currents, so that the symmetry of the curves is destroyed.

4th. The parameters of these parabolas increase directly in the ratio of the squares of the velocities of the different gales :—If x be taken as the velocity of the wind at H feet above the ground the parameter of the corresponding parabola is $\left(\frac{x^2}{H+72}\right)$ and as x varies, the parameter will vary as x^2, or as the square of the velocity of the gale.

5th. In order to render wind observations comparable, all anemometers should if possible be placed at an uniform height above the ground, and that standard height should not be lower than 20 feet above its surface but, were it generally, practicable, 50 feet, or a still greater height would be better.

6th. When it is desired to find for *small heights* the velocity V at any point H feet above the ground, from the known velocity v at a height h feet above the ground (h being *above* 15 feet), the formula is $V=v\sqrt{\frac{H+72}{h+72}}$; when H is above 50 feet above ground, the V got from the formula is slightly in excess of the actual velocity. When it is

wanted to ascertain the velocity for *great heights* above the sea level, the approximately correct formula, which is believed to be sufficiently correct for practical purposes, is $V = v\frac{H}{h}$.

7th. It would be well for meterologists to adopt this reduction formula and to express all wind velocities as referred to the height of 50 feet above the ground. This formula in this case becomes $V=v\sqrt{\frac{122}{h+72}}$."

It is no wonder after what has been given, that engineers disagree as to the pressure of wind even in a given case. We find one man taking the vertical projection of one truss, another of one and a half, another of two, and so on, and almost no reference made to any other portion of the structure or to any different projection.

Again the same pressure is taken for a bridge in a clear, open space, or for a gorge where, as Mr. Cooper (Vol. IX—394 of Transactions) has pointed out—there may be developed "local currents of much greater velocity," than the average of the storm.

Again, the same pressure is ordinarily taken for a short bridge, as for a long one, and we have a common agreement, among observers that the violence of storms is exerted over only limited areas, and is progressive in its character. There is also the other fact, that for structures of great span, the danger from repeated or "rhythmic" gusts (spoken of by Mr. Cooper) is exceedingly small. An examination of the storm record, given in the Central Park report, will show that the gusts are hardly such as to cause this danger to structures whose period of vibration is very slow.

The experiments and observations needed to secure a scientific treatment of the whole subject, and uniformity of practice among engineers, must be more exhaustive than any heretofore made; but we shall never reach the desired end by assuming that it is unreachable. In the hope that this review may assist in promoting a movement in the right direction, this paper is respectfully submitted.

P. S.—The writer would submit the following copy of a letter just received from Prof. Cleveland Abbe, of Washington, hoping that its suggestions may be favorably received by the Society.

ARMY SIGNAL OFFICE,
WASHINGTON, D. C., February 8, 1881.

MR. COLLINGWOOD,

Assistant Engineer, &c. :

SIR,—In reply to yours of the 5th inst., I will say that the relation between wind pressure and velocity has not as yet formed the subject of any special investigation by the officers of this office, as it is of minor importance in the study of weather and climate. But the practical importance of this subject, to engineers and others, is such, that this office has, on several occasions, considered the propriety of establishing at favorable stations, such as Cape Hatteras and Mount Washington, apparatus for observing simultaneously both force and velocity during gales and hurricanes.

There has always existed, however, considerable diversity of opinion as to the apparatus most appropriate to this investigation, and I should be glad to receive from the Society of Engineers, or other experts, any suggestions upon this subject.

Not having access to the bulletins of your Society, I can scarcely judge how far you have gone in the consideration of this very difficult subject, but I will append a list of titles of memoirs that you may find convenient to refer to.

I should be glad to obtain copies or titles of any memoirs relating to this subject which have been published in America.

Very respectfully yours,

CLEVELAND ABBE.

T. R. ROBINSON.—Description of an improved Anemometer, &c. Trans. Royal Irish Academy. Vol. XXII. Dublin, 1850.

G. H. L. HAGEN.—Ueber den Widerstand der Luft gegen Planscheiben, &c. Abhandlungen d. Akad. d. Winnenschaften. Berlin, 1874. An abstract also to be found in Pogg. Annalen, Vol. 152, p. 95.

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Art. VI., Wild's Repertorium für Meteorologie.

William E. Worthen.—I would ask Mr. Collingwood if he knows of any instrument by which to determine wind pressure. Some years since I had occasion for such a one, and went to Mr. James Green, the mathematical instrument maker. He showed me some, but said that they were not reliable, and that there were none manufactured that he would indorse, nor do I find since then any new form. Wind pressure is, I believe, generally estimated from velocity measures, but in pursuing my inquiry on these I consulted Charles B. Richards, one of our members, who had made experiments with two of Casella's anemometers, in his arrangements for the ventilation of the Connecticut State House; the results are published in the Journal of the Franklin Institute, May, 1875. Each instrument was tested by the maker, and a table of corrections furnished with each, and yet Mr. Richards reports, that the indications of the two meters differed so widely that little confidence could be put in them. I have never been able to arrive satisfactorily at either the plus or windward pressure on a structure, or the minus or leeward. In the matter of ventilation of houses—I take it into consideration for the furnace air-ducts in the construction of chimneys—I follow examples that have stood. There is one at Hastings-on-the-Hudson, a part of the wreck

of a sugar house, that ought, I think, to have blown down long ago. In "Silliman's Journal," some forty odd years since, I read of fowls being taken up in a whirlwind, and their feathers being blown out by the explosion of the air in their quills, showing that there was a considerable vacuum in the storm centre. Mr. Chesbrough suggests, whether there might not be a wind pressure instrument made on the principle of a pendulum, in which the the force might be estimated by the amount that the pendulum is swung out of plumb by it. It would seem that the list of a ship under canvass by the wind might offer some data of calculation, but they are unsatisfactory. M. Perrodil, in the "Annales dés Ponts et Chaussées of 1877," gives a description of a stream gauge in which the velocity of the current is determined by the torsion of a wire. I have full drawings from him, and price.